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RESPONSE OF TALL BUILDINGS WHEN SUBJECTED TO FIRE

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Abstract - *The code of practice to design the structures when subjected to fire is often not under consideration. After the events of collapse that occurred in WTC, Windsor Tower & Delft building has raised questions towards the stability of structure when subjected to fire. These incidents have shown the global collapse of the structure due to multi floor fires. This paper is dealt with the study of the response of structure at various temperatures and finally its collapse at a selected temperature using Etabs. Also the seismic analysis is made on the same structure using static pushover analysis in SAP2000.*

***Key Words*: Temperature, thermal gradient, Progressive collapse, Post earthquake fires**

1.INTRODUCTION *(Size 11 , cambria font)*

Since the collapse events of WTC, fire is considered as the major treat to building safety. Since then many researchers have given importance to the analysis of structures when subjected to fire. To reduce the loss of life and property it is important to incorporate the fire safety along with the building design. Presently the use of steel structures is increasing due to its ease of construction and structural efficiency. At the same it is the structural steel that is more easily subjected to rapid temperature variations. Therefore the fire resistance design for steel structures should become a primary aspect. From past two decades immense research has been carried out to study the structural behavior of structures under fire. The experimental full scale tests conducted at Cardington has depicted the real behavior of structural elements. On the other hand, performance based design is best adopted, as fire safety design as an integral part of structural design.

The structures subjected to fire do not only affect the stability of its components but finally ends with the collapse of the structure. Hence it can be said as fire induced progressive collapse as in the case WTC towers. Progressive collapse can be termed by a definition as a spread of primary local failure from a single component to the entire structure ultimately leading to its collapse. This happens when the structure lacks adequate continuity or ductility or redundancy to resist the failure spread. Many researches have been conducted to study the fire spread, the structural components behavior, collapse simulation and have discussed ways to strengthen the structural joints under fire circumstances. The effect of vertically travelling fires has been studied by Panagiotis et al. (2013), the response of composite structure in horizontally travelling fires has been studuied by Yaqiang Jiang et al.(2013), the collapse mechanism of steel frames when exposed to fire using implicit dynamic analysis is studied by Jian Jiang et al.(2014). The combined effect of axial force and moment yield capacity of beam-columns for thermal gradient case and uniform temperature case has been compared and studied by Maria Garlock et al.(2008). Also Sherif Yehia et al.(2013) has brought the properties of commonly used construction materials and their performance when exposed to extreme high temperatures.

1.1 Fundamental Principles

Necessitating the use of fundamental principles to understand the complex interactions in a structure lead to explain few phenomenon. Behavior of structures, under fire is affected by loss of strength caused by thermal degradation and large deflections under imposed loading. The basic relationship governing the behavior of structure subjected to fire is,

$$\varepsilon_{total} = \varepsilon_{thermal} + \varepsilon_{mechanical}$$

The unavoidable thermal strain induced in a member determines the structure's real response. This strain leads to thermal expansion which increases the length and curvature of the member. The thermal expansion induced in a structure has weak end translational restraint, strains produce displacement effective response. On the other hand thermal gradients inducing curvature leads to bowing action member ends are permitted with rotation which in turn produces deflections.

- Thermal Expansion:- It is familiar from basics that materials expand on heating. The expression for thermal expansion is given by,
  $$\varepsilon_T = \alpha\Delta T$$

The increase in temperature in a simply supported beam results in increase in length , hence total strain will be equal to thermal strain as there are no stresses developed for mechanical strain to account for.

- Thermal Bowing:- The effect of temperature gradient is significant. Higher temperatures are experienced on the exposed inner surfaces when compared with the outer surface. This induces bending in the inner surfaces due to their rapid expansion. This effect is called thermal bowing and is pre-dominantly seen in concrete and masonry structures.
- Deflections:- When a structure is under fire, it experiences thermal expansion which induces the member to increase its length. As the member cannot expand longitudinally due to end restraints, it tries to accommodate the additional length leading to large deflections.

2. BEHAVIOR OF STRUCTURAL ELEMENTS UNDER FIRE

2.1 FE Modeling:

The structural modeling and its corresponding analysis is made using Etabs 15 version. Etabs is an advanced tool to perform non linear analysis of structures. The building modeled in this software in a 30 storey composite framed structure. The structural composition consists of first 20 stories RCC framed with beams and columns. The next 10 stories consist of structural steel framed with primary and secondary beams and columns. The dimensions of the structural elements and materials are mentioned in the table 1 and table 2. A 2D plan view of the structure is shown in fig 1 and 2.

Table -1 : Material Properties

Parameter	Value
Concrete grade	M-25
Weight per unit volume	25 KN/m³
Steel grade	HYSD 500
Weight per unit volume	78.5 KN/m³
Steel Grade	Fe 345
Weight per unit volume	78.5 KN/m³

Table - 2 : Sectional Properties

Sl. No.	RCC	Dimensions (mm)
1.	Column	600 x 1200
		500 x 1000
2.	Beams	300 x 450
3.	Slab	125
		100
Sl. No.	RCC	Dimensions (mm)
1.	Column	300 x300 x13
2.	Main Beam	ISMB 300
3	Secondary Beam	ISLB 300

Fig - 1: Beam Column layout at 15th storey

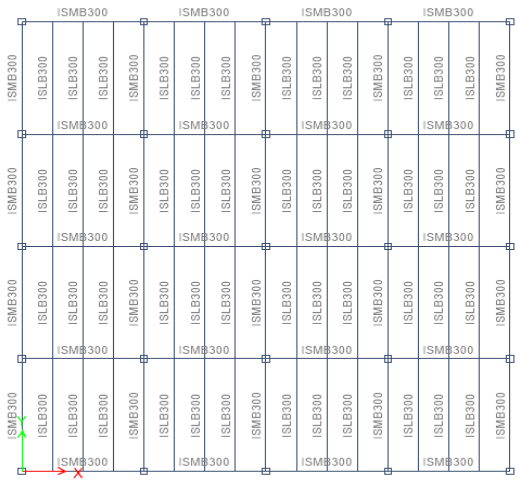

Fig - 2: Beam Column layout at 25th storey

The model is basically analyzed for static loads and load combinations. The design is checked for applied capacity. Then non linear type of analysis is done for applied temperatures. Initially the applied temperature on the top 30th floor system is 900°C. The fire is assumed to have spread horizontally and it is analyzed to study the responses. The deformed shape of the structure is shown in fig. 3.The moments due to the temperature load are observed and noted. The primary beams and columns experience huge moments when compared to secondary beams. Since the geometry of the structure is symmetrical, maximum moments are taken on one fourth portion when seen in plan. Similarly the temperature is increased by every 100°C i.e., 1000°C is applied on the same floor system and the respective moments are noted. Likewise, the above procedure is continued for every 100°C up to 1500°C noting their respective moments, deflection and axial forces. A plot of beam moments, column moments, axial force, beam deflection and joint displacements against temperature increase is shown in figures.

2.2 Discussion:

- The variation of moment v/s temperature has been observed and tabulated for beams B1, B5 & B9 and presented in chart 1. An increase in temperature from 900°C to 1500°C shows a linear increase in moments.
- As the temp spread is horizontal, the thermal expansion in beams results in lateral push in columns. Hence the column behavior is studied for moments and a graph representing temperature v/s maximum moment for selected columns is plotted as shown in chart 2. which include C1, C2, C6 & C7. It can be seen that with rise in temperature, moments too increase and maximum moment at 1500°C is experienced by C1. Therefore it can be concluded that the corner columns witness maximum lateral push.
- The variation of axial force and deformation in beams with increase in temperature are represented in chart 3 & chart 4. The beams under consideration are B1, B5 & B9. The graph shows a linear increase in axial force and deformation. In all the cases of beams it can be seen that B1 shows maximum variation with increase in temperature. Therefore it can be concluded that the peripheral beams connecting the outer columns are subjected maximum thermal environments followed by large thermal variations.

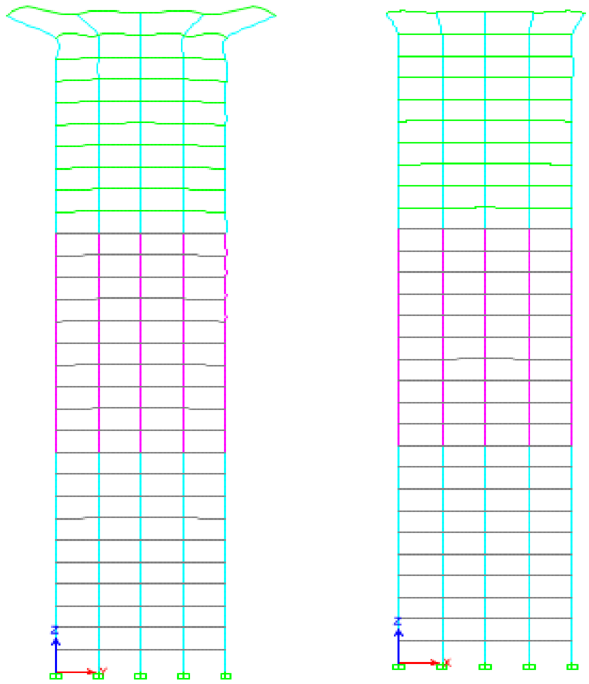

Fig - 3: Deformed Shape at different regions

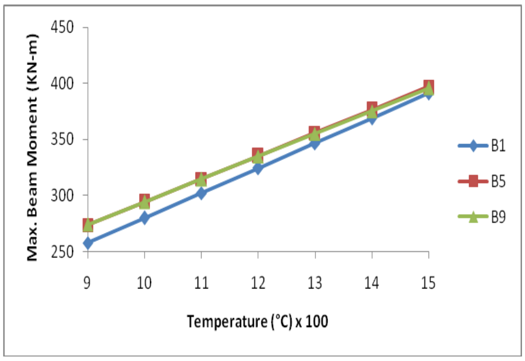

Chart - 1: Variation of beam moments

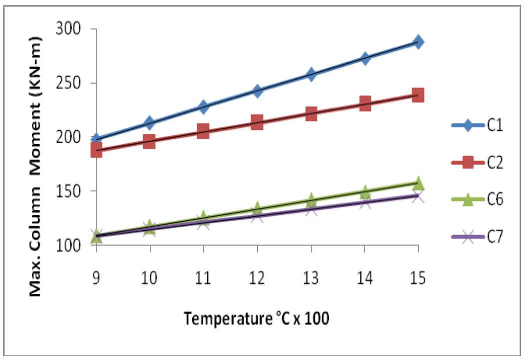

Chart - 2: Variation of Column moments

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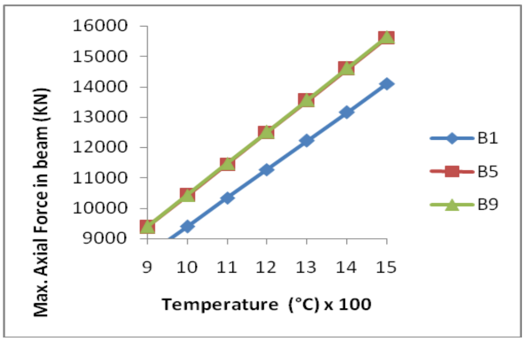


Chart - 3: Variation of axial force in beams

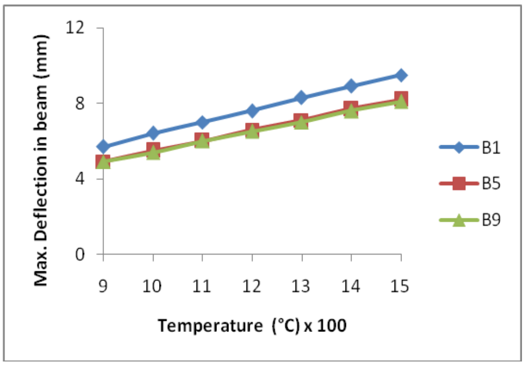


Chart - 4: Variation of max. deflection in beams

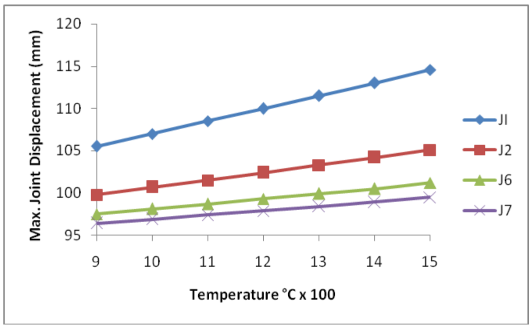


Chart - 5: Variation of max. joint displacement in x-direction

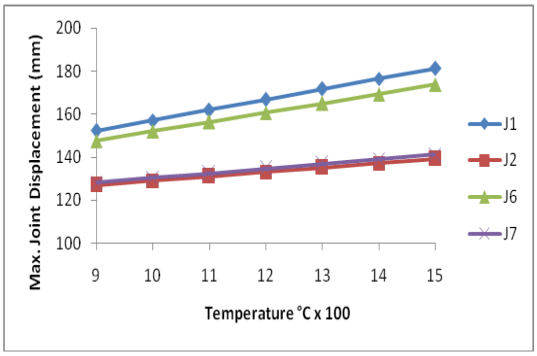


Chart - 6: Variation of max. joint displacement in y-direction

3.COLLAPSE PROCESS

Collapse of the structure is initiated once when the fire affected area reaches ultimate plastic state, where formation of hinges is one more than the degree of freedom. At this stage the structural member will not be able withstand the additional moments due to additional load acting on it, ultimately leading to its collapse. The study on the WTC towers has identified two distinct collapse mechanisms i.e., weak floor mechanism and strong floor mechanism. If the floor just below the fire affected floor reaches plastic moment capacity resulting in the formation of hinge in that floor, mechanism of weak floor collapse is initiated. If the lower adjacent floor to that of the fire affected floor is able to withstand the increasing bending moments, then the lower floor first reaches the plastic moment capacity. The hinge formation that is seen at the floor column connection symbolizes the initiation of strong floor collapse.

The plastic moments for steel sections are calculated analytically to determine the plastic moment capacity of the member. The plastic moment for rectangular hollow column section is 554.5KN-m and that for I-section beam is found to be 257.73Kn-m. The moments in the members those who have exceeded plastic moment show yielding of the sections and formation of considerable number of hinges. Here beams B1, B5 & B9 and their corresponding similar beams exceed the full plastic moment values. Hence the horizontal floor system is expected to fail and collapse on the lower floor due to inadequate continuity in the members. The moments obtained from numerical investigation is tabulated in table 3. The collapse process and the corresponding deformation shapes are shown in fig. 4.

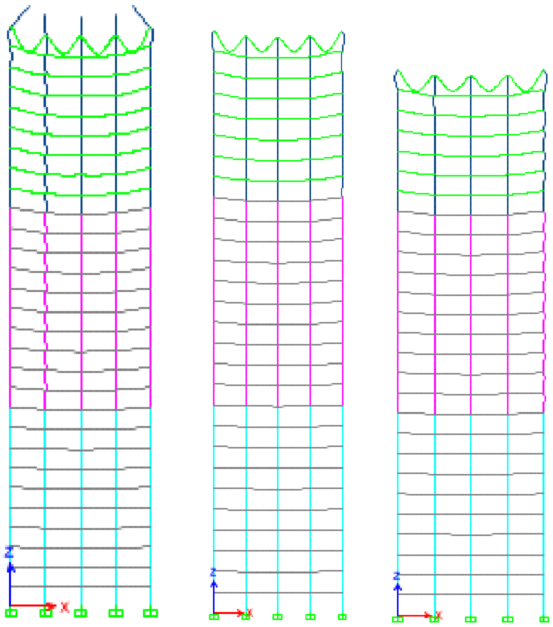
3.1Discussion

Due to continuous heating of 30th floor, the beams are in a state wherein they can no more withstand the thermal loads. The floor system experiences large deflections and moments and also the number of hinges exceed the number of degrees of freedom of the system. At this point the beams of 30th floor yield completely and collapse on to the lower 29th floor. As seen in fig. 4(a) the beams have collapsed and columns have buckled as a result of heavy loads.

The 29th floor system witnesses huge moments and deflections, due to the collapse of the whole 30th floor system. The beams of 29th floor do not allow redistribution of loads, as a result, deflection increases. This deflection in beam makes the joint restraints weak at beam column joint leading to buckling of columns. Once the joint become weak and loads do not take alternate paths, on the other hand number of plastic hinges increase at joints and at location of maximum bending moments, ultimately leads to collapse of floor system along with columns. This process of yielding and collapsing continues on each floor finally leading to the collapse of entire structure. Fig. 4(b) & (c) shows the process of collapse in 29th & 28th floor. Such a collapse can be classified as progressive collapse.

Table - 3: Moments of Beams and columns

Beam No.	Mom KN-m	Col. No.	Mom KN-m
B1	257.77	C1	197.98
B2	70.43	C2	187.48
B5	273.65	C3	175.17
B6	87.77	C6	109.42
B9	274.25	C7	108.88
B10	86.9	C8	100.28



(a) (b) (c)

Fig - 4: Collapse Process

4. COLLAPSE DUE TO POST EARTHQUAKE FIRES

Ground vibrations caused by earthquakes, fosters annihilating destruction to the structures. Occasionally earthquakes are superseded by fire in the building due to electric short circuit or gas explosion or presence of harmful fuels which seem to be more hazardous than earthquake alone. Therefore their occurrence during earthquake can cause the collapse of entire structure because of additional stresses due to fire. Generally with availability of design codes for earthquake seismic design of structure is done without fire consideration. In the present analysis both these events are expected to occur separately. Earthquake analysis for a structure can be done using either linear or non-linear methods. Here structure is analyzed using Non-Linear Static Pushover Analysis.

Pushover analysis is a simple computer based technique and widely used method for performance based design of structures subjected to seismic loadings. Due to its simplicity & effectiveness in determining the forces & deformations, has gained notable importance from past two decades. The analysis is made on a 30 storey composite framed structure using SAP2000 software tool. Due to symmetry in geometry, the analysis is considered in only one direction. The modeling techniques follow simple analysis tools. Initially the structure is modeled and analyzed for linear analysis and also checked designed for the same. Once the analysis and design is complete showing the structure to be safe, non linear hinges are assigned to structural members to undergo plastic deformation. Bi-axial moment with axial force hinge is assigned to all columns. Moment along major axis hinge is assigned to all beams. The hinge properties are calculated as per FEMA356 guidelines. Later, the necessary input for pushover analysis is fed into the software and the analysis is solved to obtain static pushover curve and moment-curvature relation for a particular hinge.

4.1Results

After conducting various pushover runs, formation of hinges were observed at various points which are considered to be obnoxious from design point of view. Hence the location where hinge formation is seen requires additional strengthening. Fig. 5 shows an elevation view of the structure with formation of hinges. The hinge properties showing the safety of the structure or the condition of the structure under the influence of lateral loads is shown in fig. 6. The five points in the figure named A, B, C, D and E define the deformation behavior of plastic hinges. The other three points named IO (Immediate Occupancy), LS (Life Safety) and CP (Collapse Prevention) define the acceptance criteria. In this structure considerable number of hinges have just crossed collapse prevention stage, hence can be described to have reached the collapse level. The hinge result for hinge no. 2811H1 showing a plot of moment v/s plastic rotation is shown in fig. 7.

The capacity of a structure is a representation of nonlinear force against its displacement which is commonly called as static pushover curve. The plot of the same for the structure under consideration is shown in fig. 8. A comparison of the capacity of the structure with that of the ground motions acting on it is graphically represented by Capacity Spectrum. The base shear and displacements are converted into spectral values that are defined by spectral acceleration and spectral displacement by means of effective modal masses and modal participating factors. A plot of the same is shown in fig. 9.

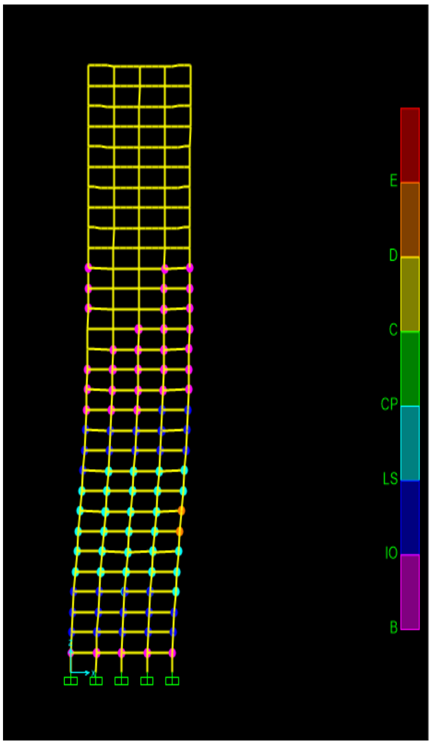


Fig - 5: Formation of Hinges in structure

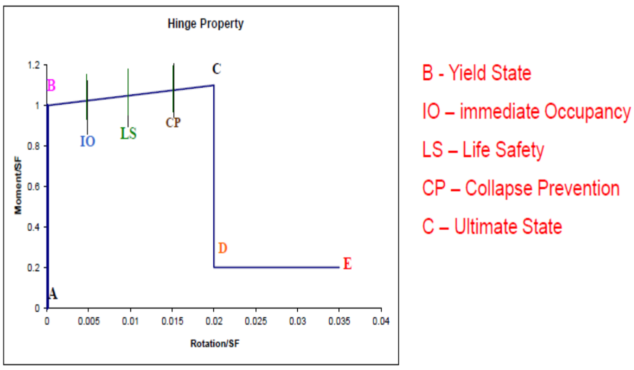


Fig -6: Hinge Properties

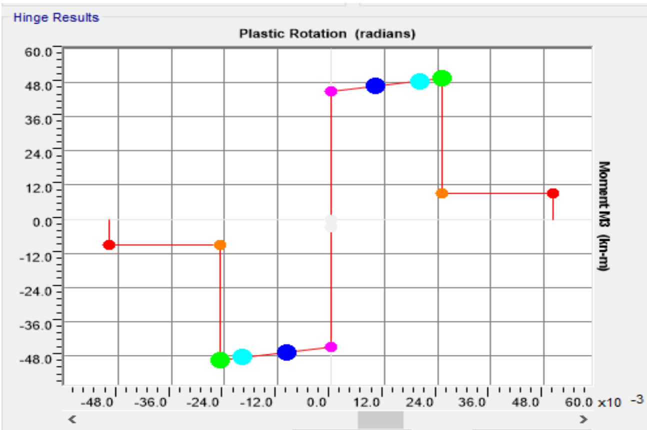


Fig - 7: Hinge result representing moment-curvature relation

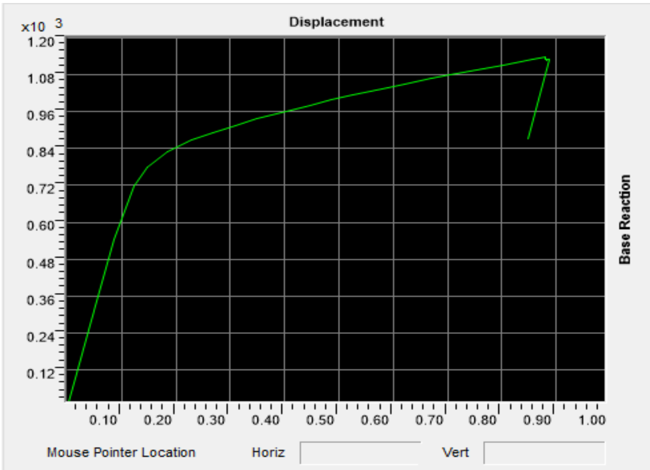


Fig - 8: Static Pushover Curve

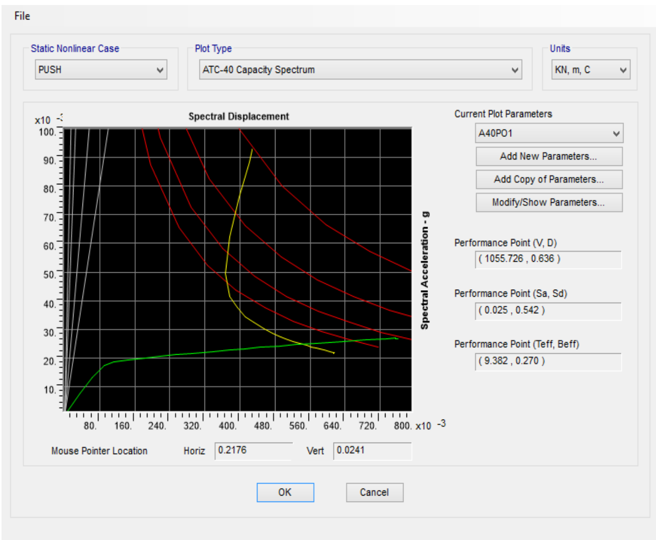


Fig - 9: Capacity & Demand Spectrum

4.2. Discussion & Conclusions

The static pushover curve for a 30 storey building is shown in the fig. 8. The curve is found to be linear in the elastic region and slowly gets deviated from linearity as the structural members undergo plastic deformation. Even after entering into the plastic region, the curve is found to be linear again with reducing slope. The curve is said to exhibit a bilinear configuration.

The performance level of the structure is obtained from capacity and demand spectrum which is represented in the fig. 9. It can be incurred that the demand and the capacity curve tend to intersect each other in the life safety level of the structure. At this instance it can be seen that significant amount of initial stiffness is lost but however gives space for additional deformation before the structure could collapse.

The basic performance level of the structure is not satisfied for earthquake case. A few hinges as shown in fig. 11 have been developed due to earthquake and are found to have reached ultimate state of collapse. This shows that the structure will not withstand the accidents due to post earthquake fire. Hence the joints indicated with plastic hinges require additional ductile strengthening not only to perform better during earthquake but also to withstand the huge damage and loss caused due to fire following earthquakes.

5. CONCLUSIONS

All structures either small or large in base dimensions, tall or short in height, regular or irregular in geometry are subjected to few common criteria which affect the stability of the structure. One such criterion that is considered harmful to the structure is studied in this thesis i.e., structure subjected to fire. In this thesis the fire action on a tall building is studied by considering a 30storey composite structure with top 10 stories designed as structural steel components. The analysis is made by considering the fire explosion at the top most floors. The various responses of the structure are noted by varying the temperature from 900 degrees to 1500degree Celsius. This thesis covers the variation of beam moments, column moments, maximum axial force and deflection in beams as well as the variation of maximum joint displacements along both x and y directions against each temperature.

It can be concluded that all of the above mentioned parameters show linearly increasing plots with temperature increase for steel sections. Also it can be seen that the beams connecting with the outer perimeter columns show maximum displacements as they are subjected to large thermal environments. Hence these structural components should be first prioritized for structural stability.

This research work has also made a study on the collapse mechanism of the tall structure when it is subjected to a temperature of 900°C. The location of formation of plastic hinges is noted and the increase in load enhances the collapse of such beams leading to the fall down of entire floor system. This furthermore doubles the load on the lower floor system, increasing the moments and reducing the stiffness of the floor system. In return due to loss of stability, the floor system collapses along with columns. The continuation of the process progressively leads to its collapse.

Furthermore the thesis has also focused on the seismic analysis of the structure subjected post earthquake fire. The analysis is performed using static non linear pushover analysis. The report has covered the procedure for FE modeling and pushover method assigning technique using SAP2000. A plot of the results obtained from the analysis is discussed. The static pushover curve showing a plot of displacement against base reaction is shown. The performance point of the structure is obtained form a plot of capacity and demand curve. It can be incurred from the plot that the performance of the structure requires additional ductile strengthening to withstand both earthquake vibrations and also post earthquake fire.

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